Evaluation of Residual Drift Demands in Steel Frames Subjected to Narrow-Band Earthquake Ground Motions

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SUMMARY:
In this study, residual inter-story drift demands in steel buildings designed with a modern seismic code when subjected to narrow-band earthquake ground motions are estimated. For this purpose, four steel buildings with different number of stories were designed accordingly to the Mexico City Seismic Code. The frame models were subjected to a set of 30 narrow-band earthquake ground motions recorded on stations placed in soft soil sites of Mexico City. All the earthquake ground motions were scaled to reach several levels of intensity in order to perform incremental dynamic analyses. Thus, results were statistically processed to obtain fragility curves of peak and permanent drift demands and, subsequently, hazard curves of both types of drift demands for each frame model. It is observed that the steel structures that might undergo peak drift limits associated to the collapse prevention results in unrecoverable large permanent drifts, when subjected to narrow-band earthquake ground motions.

Keywords: Residual inter-story drift; narrow-band ground motions; steel frames

1. INTRODUCTION

Nowadays, recently proposed performance-based seismic design and assessment procedures for new and existing structures emphasize on the estimation of peak (transient) lateral drift demands. However, earthquake field reconnaissance have evidenced that permanent (residual) lateral displacement demands after earthquake excitation (e.g. residual roof drift ratio or maximum residual inter-story drift ratio) also play an important role in defining the seismic performance of a structure and it can have important consequences. For instance, several dozen damaged reinforced concrete (RC) buildings in Mexico City had to be demolished after the 1985 Michoacan earthquake because of the technical difficulties to straighten and repair buildings with large permanent drifts (Rosenblueth and Meli 1986). Years later, Okada et al. (2000) reported that several low-rise reinforced concrete (RC) buildings suffered light structural damage but experienced relatively large residual deformations as a consequence of the 1995 Hyogo-Ken Nambu earthquake even though they had sufficient deformation capacity. In addition, a recent field investigation in Japan highlighted that a residual inter-story drift of about 0.5% is perceptible for building occupants and a residual inter-story drift of about 1.0% could cause human discomfort (McCormick et al. 2008). These field observations imply that the total expected economic losses computed from peak drift demands and peak floor acceleration demands could be smaller than those computed taking into account permanent drift demands due to necessity of demolishing structures having excessive permanent deformations although they did not experience severe structural damage (Miranda and Ramirez 2010). Therefore, several researchers have highlighted that the estimation of residual drift demands should also play an important role during the design of new buildings (e.g. Pampanin et al. 2003, Erochko et al. 2011) and the evaluation of the seismic structural performance of existing buildings (e.g. Ruiz-García and Miranda 2005, 2006, 2010 Uma et al. 2008, Yazgan and Dazio 2008).

Motivated by earthquake field reconnaissance observations, researchers have performed analytical investigations aimed at gaining further understanding on the parameters that influence the amplitude and height-wise distribution of residual drift demands in existing multi-story buildings (Pampanin et al. 2003, Erochko et al. 2011, Ruiz-García and Miranda 2005, 2006, Uma et al. 2008, Yazgan and Dazio 2008). They have reported that the residual drift demand amplitude and distribution over the height depends on the component hysteretic behavior (Pampanin et al. 2003, Ruiz-García and Miranda 2005, 2006), building frame mechanism, structural overstrength (Ruiz-García and Miranda 2005, 2006) as well as the ground motion intensity (Pampanin et al. 2003, Ruiz-García and Miranda 2005, 2006, Uma et al. 2008).

However, it should be noted that previous studies have employed earthquake ground motions recorded in accelerographic stations placed on rock or firm sites, which have very different frequency content and duration characteristics than those recorded in stations placed on soft soil sites (e.g. the lake-bed zone of Mexico City, or the Marina district in San Francisco Bay). These types of records are characterized by relatively long predominant period of the ground motion and low-frequency content (i.e. narrow-band earthquake ground motions). Furthermore, a recent study (Erochko et al. 2011) showed that ductile steel moment resisting frames designed under seismic loading following the ASCE 7-05 standard, which is the up-to-date standard for seismic design in the United States, would experience excessive residual inter-story drifts when subjected to earthquake ground motions scaled to reach the Maximum Credible Excitation level. Therefore, there is a need to evaluate the expected residual inter-story drift levels that buildings designed with modern seismic provisions for soft soil sites would experience and, furthermore, to evaluate if designing for drift limits associated to collapse prevention would lead to unrecoverable permanent drifts.

The objective of this paper is to present the results of the evaluation of residual inter-story drift demands in steel buildings designed with a modern seismic code when subjected to narrow-band earthquake ground motions. For this purpose, four steel buildings with different heights designed accordingly to the 2004 Mexico City Seismic Design Provisions (MCSDP) were evaluated under a set of earthquake ground motions recorded on stations placed in soft soil sites of Mexico City during 5 historical earthquakes.

2. BUILDING FRAME MODELS

For the estimation of residual inter-story drift demands four regular moment-resisting steel frames having 4, 6, 8 and 10 stories were considered. The frames are denoted F4, F6, F8 and F10, respectively. The frames which were assumed to be used for office occupancy have three bays of 8 meters and inter-story heights of 3.5 meters as illustrated in Fig. 2.1. It should be noted that the study case frames were designed by a structural engineering office. Each frame was provided with ductile detailing and its lateral strength was established according to the MCSDP. A36 steel was used for the beams and columns of the frames.

A two dimensional, lumped plasticity nonlinear model of each frame was prepared and analyzed with the program RUAUMOKO (Carr 2008). For this purpose, a-non-degrading elasto-plastic moment-curvature relationship with 3% strain-hardening was used to represent the cyclic behavior of the steel members. Flexural moment capacity of beams and columns was determined from a nominal steel yield stress of 2530 kg/cm^2. In addition, slab contribution to the beam's bending capacity was neglected in this study. While the slabs were modeled as rigid in-plane diaphragms, the columns in the first story were modeled as clamped at their bases. Second order effects were explicitly considered. Time-history analyses were carried out for each frame subjected to a set of narrow-band earthquake ground motions described below. Critical damping ratio was assumed equal to 3%. Relevant characteristics for each frame, such as the fundamental period of vibration (T_1), and the seismic coefficient and displacement at yielding (C_y and D_y) are shown in Table 2.1 (the latter two values were established from pushover analysis).

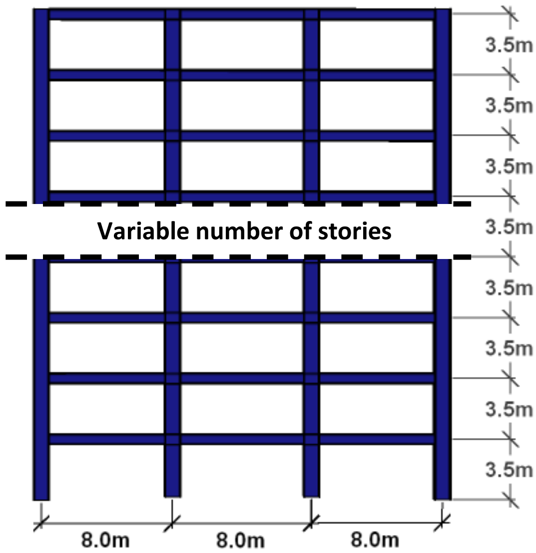


Figure 2.1. Geometrical characteristics of moment-resisting steel frames

Table 2.1. Structural properties of steel frames under consideration

Frame	Number of Stories	T_1 (s)	C_y	D_y (m)
F4	4	0.90	0.45	0.136
F6	6	1.07	0.42	0.174
F8	8	1.20	0.38	0.192
F10	10	1.37	0.36	0.226

3. SELECTION OF NARROW-BAND EARTHQUAKE GROUND MOTIONS

The regular steel frames were subjected to a set of 30 narrow-band and long duration ground motions recorded at soft soil sites of Mexico City. It should be recall that most of the structural damage occurred during the September 19, 1985 Michoacan earthquake were recorded in the sites considered in this investigation. The records were taken from sites having soil periods of two seconds, and recorded during seismic events with magnitudes near of seven or larger and having epicenters located at distances of 300 km or more from Mexico City. It should be mentioned that sites having soil periods of two seconds are fairly common within the Lake Zone, and that the higher levels of shaking in terms of peak ground acceleration have been consistently observed at these sites. Some important characteristics of the records are summarized in Table 3.1. In this table, while *PGA* and *PGV* stand for the peak ground acceleration and velocity, t_D is the strong-motion duration estimated according to the Trifunac and Brady (1975) criterion, which is defined as the time interval delimited by the instants of time at which the 5% and 95% of the Arias Intensity occurs. Note that the average duration of the records equals 74.4 sec.

Table 3.1. Selected narrow-band ground motions considered in this study

Records	Date	Magnitude	Station	*PGA* (cm/s²)	*PGV* (cm/s)	t_D *(s)*
1	19/09/1985	8.1	SCT	178.0	59.5	34.8
2	21/09/1985	7.6	Tlahuac deportivo	48.7	14.6	39.9
3	25/04/1989	6.9	Alameda	45.0	15.6	37.8
4	25/04/1989	6.9	Garibaldi	68.0	21.5	65.5
5	25/04/1989	6.9	SCT	44.9	12.8	65.8
6	25/04/1989	6.9	Sector Popular	45.1	15.3	79.4
7	25/04/1989	6.9	Tlatelolco TL08	52.9	17.3	56.6
8	25/04/1989	6.9	Tlatelolco TL55	49.5	17.3	50.0
9	14/09/1995	7.3	Alameda	39.3	12.2	53.7
10	14/09/1995	7.3	Garibaldi	39.1	10.6	86.8
11	14/09/1995	7.3	Liconsa	30.1	9.62	60.0
12	14/09/1995	7.3	Plutarco Elías Calles	33.5	9.37	77.8
13	14/09/1995	7.3	Sector Popular	34.3	12.5	101.2
14	14/09/1995	7.3	Tlatelolco TL08	27.5	7.8	85.9
15	14/09/1995	7.3	Tlatelolco TL55	27.2	7.4	68.3
16	09/10/1995	7.5	Cibeles	14.4	4.6	85.5
17	09/10/1995	7.5	CU Juárez	15.8	5.1	97.6
18	09/10/1995	7.5	Centro urbano Presidente Juárez	15.7	4.8	82.6
19	09/10/1995	7.5	Córdoba	24.9	8.6	105.1
20	09/10/1995	7.5	Liverpool	17.6	6.3	104.5
21	09/10/1995	7.5	Plutarco Elías Calles	19.2	7.9	137.5
22	09/10/1995	7.5	Sector Popular	13.7	5.3	98.4
23	09/10/1995	7.5	Valle Gómez	17.9	7.18	62.3
24	11/01/1997	6.9	CU Juárez	16.2	5.9	61.1
25	11/01/1997	6.9	Centro urbano Presidente Juárez	16.3	5.5	85.7
26	11/01/1997	6.9	García Campillo	18.7	6.9	57.0
27	11/01/1997	6.9	Plutarco Elías Calles	22.2	8.6	76.7
28	11/01/1997	6.9	Est. # 10 Roma A	21.0	7.76	74.1
29	11/01/1997	6.9	Est. # 11 Roma B	20.4	7.1	81.6
30	11/01/1997	6.9	Tlatelolco TL08	16.0	7.2	57.5

4. SEISMIC PERFORMANCE OF STEEL FRAMED BUILDINGS

The seismic performance of the selected steel frames is estimated in terms of peak and residual drift demands. In first place, incremental dynamic analysis (Vamvatsikos and Cornell 2002) is used to assess the seismic performance of the steel frames under narrow-band motions at different intensity levels. Next, the well-known seismic performance-based assessment procedure suggested by the Pacific Earthquake Engineering Center (Cornell and Krawinkler 2000, Dierlein 2004) in the United States is applied, which indicates that the mean annual frequency of exceedance of an engineering demand parameter (*EDP*) of interest exceeding a certain level *edp* can be computed as follows:

$$\lambda(EDP > edp) = \int_{IM} P\left[EDP > edp \mid IM = im\right] \cdot \left| d\lambda_{IM}(im) \right| \tag{4.1}$$

where *IM* denotes the ground motion intensity measure, $P[EDP>edp \mid IM=im]$ is the conditional probability that an *EDP* exceeds a certain level of *edp* given that the *IM* is evaluated at the ground motion intensity measure level *im*. In addition, $d\lambda_{IM}(im)$ refers to the differential of the ground motion hazard curve for the *IM*. In this context, while the first term in the right-hand side of Eqn. 4.1 can be obtained from probabilistic estimates of the *EDP* of interest, the second term in Eqn. 4.1 is representing by the seismic hazard curve, which can be computed from conventional Probabilistic Seismic Hazard Analysis (PSHA), evaluated at the ground motion intensity level *im*. Note the importance of the ground motion intensity measure for assessing the structure's seismic performance, which is the joint between earthquake engineering and seismology. Particularly, three features are desirable in a selected ground motion intensity measure: sufficiency, efficiency, and scaling robustness (Bazzurro 1998, Shome 1999, Luco 2002, Iervolino and Cornell 2005). Sufficiency means that given an *IM* the structural response is insensitive to other ground motion parameters (e.g. magnitude and distance). Efficiency is defined as good explanatory power of the *IM* with respect to some *EDP*; this may help in reducing the record-to-record variability to estimate the structural response in such a way that it is one of the most important features of an *IM*. Finally, robustness means that the amplitude (linear) scaling of records does not induce bias in the estimation of the seismic demand. In the present study, the spectral acceleration at first mode of vibration $Sa(T_1)$ was selected as *IM*. It is important to point out that the records used herein allow the use of a scaling criteria based on $Sa(T_1)$: A) First, due to sufficiency of $Sa(T_1)$ with respect to magnitude and distance; B) Second, due to the similar spectral shape of the records, because the ground motion records selected have similar values of the parameter N_p (see definition below) proposed by Bojórquez and Iervolino (2011), which is observed in Fig. 4.1, where the response spectra of the records scaled for similar values of $Sa(T_1)$ for a period of T=0.90s (the fundamental period of frame F4) are shown; and C) The property known as scaling robustness is satisfied, and this is valid although significant bias usually occurs when increasing nonlinear structural behavior. Bojórquez and Iervolino (2011) demonstrated that for scale factors in a range of 1 to 100, no significant bias occurs for important levels of nonlinear behavior (ductility demands up to six) if the records are selected with similar values of N_p. Note that N_p is defined in Eqn. 4.2, where $Sa_{avg}(T_1,...,T_N)$ represents the geometrical mean between the periods T_1 and T_N. Further information of this parameter is provided in Bojórquez and Iervolino (2011).

$$N_p = \frac{Sa_{avg}(T_1,...,T_N)}{Sa(T_1)} \tag{4.2}$$

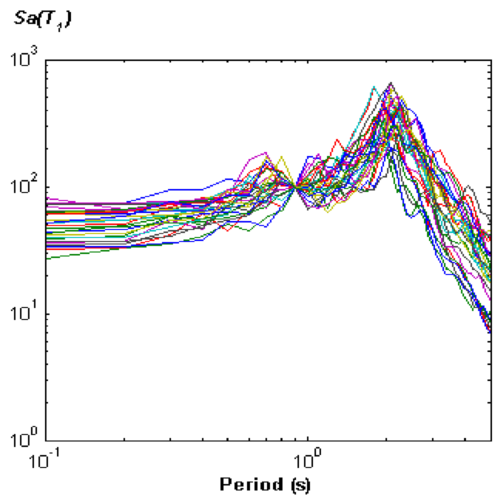


Figure 4.1. Elastic response spectra for the records scaled at the same spectral ordinate $S_a(T_1)$=100 cm/s² for *T* equals 0.9s and 3% of critical damping

When $Sa(T_1)$ is selected as *IM*, Eq. (4.1) can be expressed as:

$$\lambda(EDP > edp) = \int_{S_a(T_1)} P\left[EDP > edp \mid S_a(T_1) = s_a\right] \cdot \left|d\lambda_{S_a(T_1)}(s_a)\right| \tag{4.3}$$

where $d\lambda_{S_a(T_1)}(s_a) = \lambda_{S_a(T_1)}(s_a) - \lambda_{S_a(T_1)}(s_a + ds_a)$ is the hazard curve differential expressed in terms of $S_a(T_1)$. Eqn. 4.3 was used to evaluate the structural reliability of the steel frames in terms of two *EDPs*: peak and residual inter-story drift demands. Herein, a lognormal distribution is considered to estimate $P\left[EDP > edp \mid S_a(T_1) = s_a\right]$, for this case the probability that *EDP* exceeds *edp* given $S_a(T_1)$ is given by:

$$P(EDP > edp \mid S_a(T_1) = s_a) = 1 - \Phi\left(\frac{\ln x - \hat{\mu}_{\ln EDP|S_a(T_1)=s_a}}{\hat{\sigma}_{\ln EDP|S_a(T_1)=s_a}}\right) \tag{4.4}$$

In Eqn. 4.4, $\hat{\mu}_{\ln EDP|S_a(T_1)=s_a}$ and $\hat{\sigma}_{\ln EDP|S_a(T_1)=s_a}$ are the sample mean and standard deviation for the *EDP*, respectively, and $\Phi(\cdot)$ is the standard normal cumulative distribution function. While maximum inter-story drift has been found to be well represented by a lognormal distribution (Shome 1999, Aslani and Miranda 2003), a Kolmogorov-Smirnov test was developed to validate the use of this distribution for the case of residual inter-story drift demands, as Ruiz-García and Miranda (2010) suggest this probability density function is representative of residual inter-story drift demands.

5. NUMERICAL RESULTS OF PEAK AND RESIDUAL DEMANDS

The peak and residual drift demands are compared in this section, first, at different intensity levels in terms of spectral acceleration, and secondly, the demand hazard curves are illustrated. Fig. 5.1 shows the structural response in terms of median peak inter-story drift ratio, IDR_{max}, and median maximum residual inter-story drift ratio, $RIDR_{max}$, for all the frames analyzed. From Fig. 5.1, it can be seen that values of IDR_{max} and $RIDR_{max}$ increases as the ground motion intensity measure tends to increase. Furthermore, it can also be seen that the IDR_{max} equals 3%, which is the allowable peak inter-story drift to avoid collapse recommended by 2004 Mexico City Seismic Design Provisions, when $Sa(T_1)$ is in a range from 0.8g up to 1.1g for all the steel frames. In addition, Fig. 5.2 shows the ratio of median residual inter-story drift demands divided by median peak inter-story drift demands for all frames and intensity levels. It can be observed that this ratio could be in a range from 15% to 18% for the F4 and F6 models when the frames also reach peak inter-story drift ratio demands in the order of 3%, which indicates the importance of taking into account the influence of residual deformations during seismic assessment.

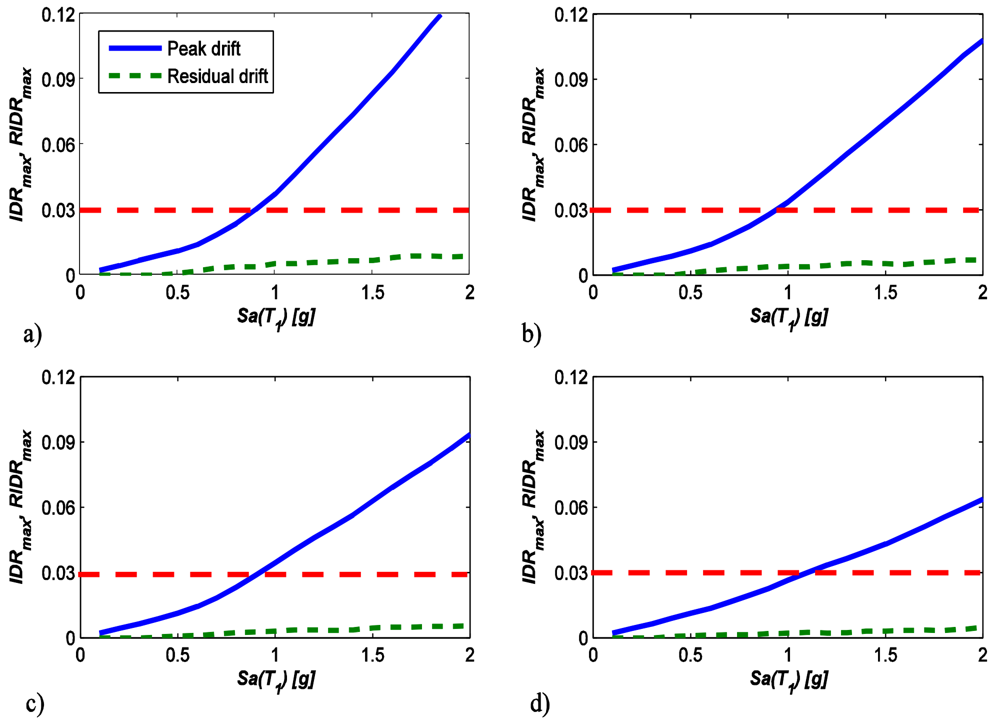


Figure 5.1. Median values of IDR_{max} and $RIDR_{max}$ using nonlinear incremental dynamic analysis for frame: a) F4, b) F6, c) F8, d) F10

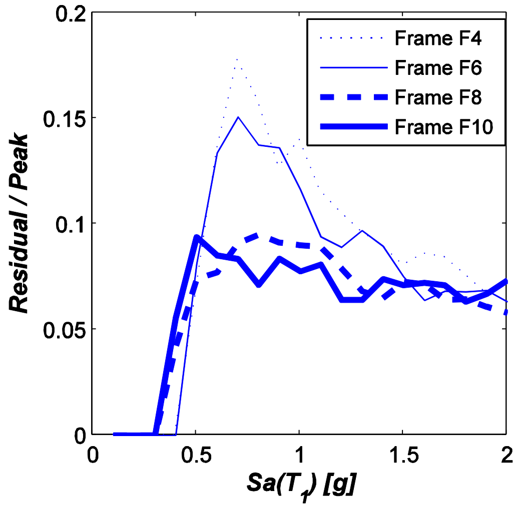


Figure 5.2. Ratio of median $RIDR_{max}$ divided by median IDR_{max} at different intensity levels in terms of $Sa(T_1)$ for all frames under consideration

To further illustrate the importance of residual inter-story drift demands, Fig. 5.3 compares the ratio of residual and peak inter-story drift demands for frame F4 obtained for each record under consideration. An important observation that can be made is that for this intensity level, values up to 0.5 are observed for some records in the ratio of residual and peak response. For instance, the ratio of residual divided by peak response is around 50% for records number 3 and 30. This observation means that if IDR_{max} is equals 0.03, the $RIDR_{max}$ would be 0.015, which is more than three times beyond the residual inter-story drift of 0.5% suggested by McCormick (2008) as the target limit for structural demolition. It suggests the importance to recognize the use and control of residual inter-story drift as performance parameter for seismic design of structures.

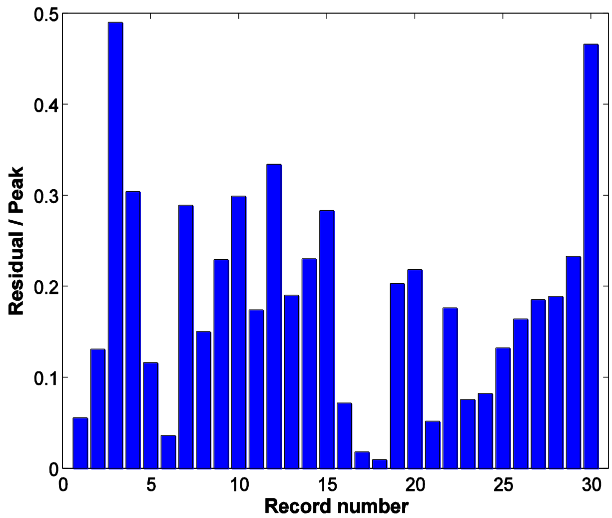


Figure 5.3. Ratio of $RIDR_{max}$ and IDR_{max} at $Sa(T_1)$=1g for frame F4 and for all the records under consideration

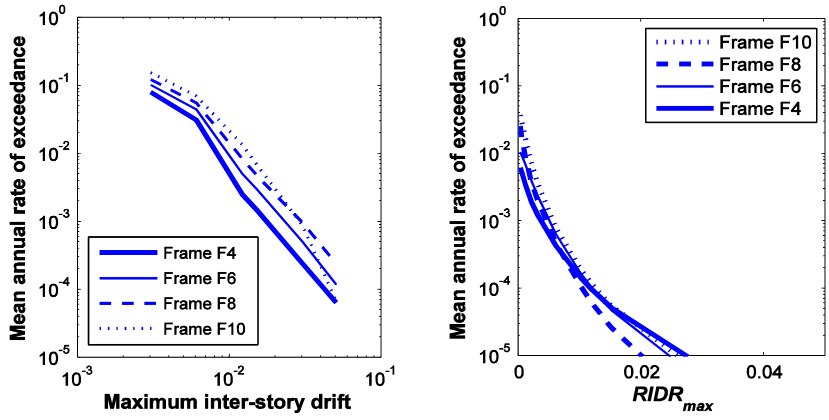
The numerical assessment of the mean annual rate of exceeding IDR_{max} and $RIDR_{max}$ for the selected frames subjected to the thirty narrow-band ground motion records is addressed in this section. To this task, the ground motion hazard curves corresponding to the *Secretaría de Comunicaciones y Transportes* (SCT) site in Mexico City established by Alamilla (2001) were employed. Fig. 5.4 shows the seismic hazard curves for the four frames analyzed in terms of the mean annual rate of exceeding IDR_{max} and $RIDR_{max}$. For both cases analyzed, the curves are quite similar for all the frames, suggesting that although the frames designed according the MCSDP were not originally developed for specific structural reliability values, the structural designs obtained with this seismic code tend to have similar annual rates of exceeding the IDR_{max} which is the main parameter used by the Mexico City Seismic Design Provisions to satisfy the earthquake resistant design criteria of buildings. Furthermore, the results illustrate that also the $RIDR_{max}$ seismic hazard curves, or the structural reliability in terms of residual inter-story drift demands, is practically the same for all the frames analyzed. Note that according with McCormick (2008) as it was mentioned before; a residual inter-story drift in a range about 0.5-1.0% is perceptible for building occupants and could cause human discomfort, in such a way that the structural reliability of the steel frames if $RIDR_{max}$ is used as performance parameter would be similar, at least for the selected steel frames subjected to narrow-band earthquake ground motions.

A direct comparison of seismic hazard curves of peak and residual inter-story drift demands for each frame is shown in Fig. 5.5, which allows a better seismic performance assessment of the frames. Firstly, the IDR_{max} and $RIDR_{max}$ seismic hazard curves for frame F4 are shown in Fig. 5.5a. It is observed that for a target IDR_{max} equals to 3%, the residual inter-story drift demand is about 0.8% in order to have the same annual rate of exceedance of both parameters. Likewise, frame F6 could experience $RIDR_{max}$ equals to 0.65% to have the same mean annual rate of exccedance corresponding to a IDR_{max} equal to 3.0%. Therefore, although the inter-story drift threshold established for the Mexican seismic regulations guarantee a good performance and could avoid the structural collapse of buildings, it cannot warrant the demolition of the building after the earthquake because the residual inter-story drift is larger than the 0.5% residual drift limit highlighted in Rosenblueth and Meli (1986). In the case of frames F8 and F10, for a mean annual rate of exceeding about 0.001 (i.e. return period of 1000 years) the IDR_{max} is equals to 3% and $RIDR_{max}$ is around 0.5%. In conclusion it is deemed necessary to establish new target limit for collapse prevention in the Mexican design provision to avoid the demolition of a building, or alternatively the inclusion of new performance parameters to satisfy the same structural reliability in terms of peak and residual inter-story drift demands.

6. CONCLUSIONS

The main results of the evaluation of residual and peak inter-story drift demands in steel buildings designed with a modern seismic design code when subjected to narrow-band earthquake ground motions records are presented in this paper. For this purpose, four steel buildings with different heights designed accordingly to the 2004 Mexico City Seismic Design Provisions were evaluated under a set of earthquake ground motions recorded on stations placed in soft soil sites of Mexico City. Firstly, results of incremental dynamic analyses showed that the ratio of residual divided by peak response can be as large as 50% for a 4-story building, indicating that if IDR_{max} is equals 3%, the $RIDR_{max}$ would be 1.5%, which is more than three times beyond the residual inter-story drift of 0.5% as the target limit for structural demolition.

The numerical results of the hazard curves in terms of peak lateral drift demand suggests that, although the frames designed with the Mexico City building code were not designed for a specific annual rate of exceeding some level of inter-story drift demand, the structural reliability is almost similar for all the structures here analyzed. Consequently, the designs obtained with this code give place to similar safety levels. By comparison of peak and permanent drift demand hazard curves, it is shown how the steel structures designed for peak drift limits associated to the collapse prevention could experience unrecoverable large permanent drifts when subjected to narrow-band earthquake ground motions, in such a way that the structures may need to be demolished. It suggests the importance to recognize the use and control of residual inter-story drift as performance parameter for seismic design of structures in addition to peak inter-story drift demand. Finally, permanent lateral drift demand levels found in this investigation suggest that inclusion of permanent drift limits in future guidelines of seismic design codes is crucial to improve the seismic structural performance of buildings under narrow-band seismic ground motions, hence to reduce the economic losses and consequences of the earthquakes.



a) **Peak demands** b) **Residual demands**

Figure 5.4. Seismic hazard curves of peak and residual inter-story drift demands for steel frames considered in this investigation

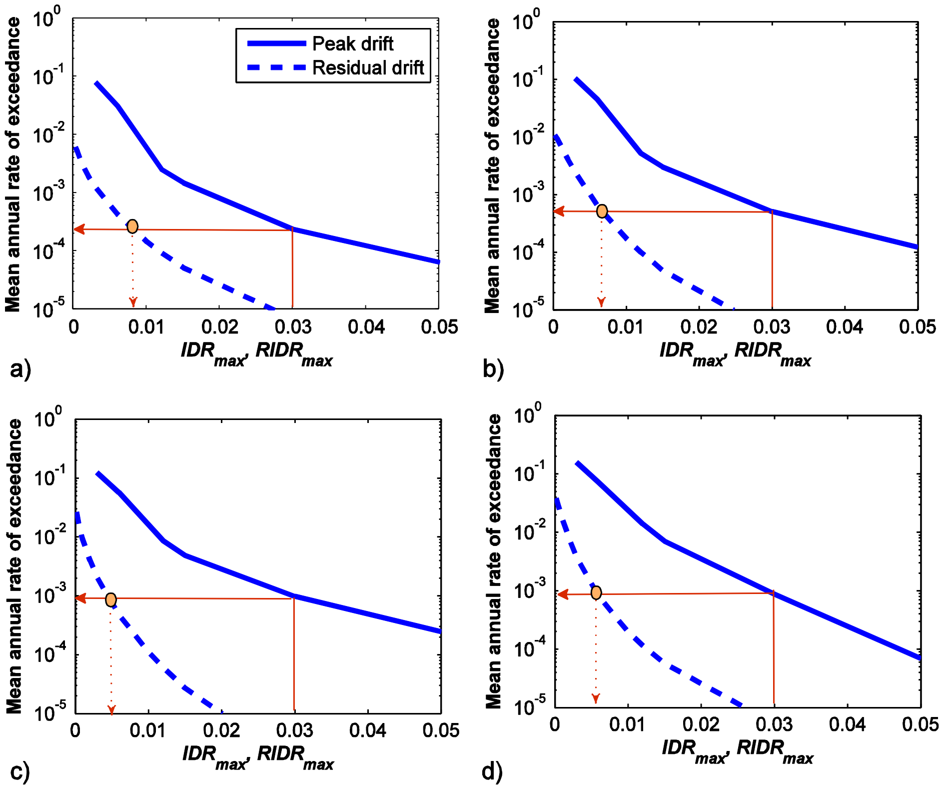


Figure 5.5. Comparison of seismic performance of the steel frames for peak and residual inter-drift demands for frame: a) F4, b) F6, c) F8 and d) F10

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